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HILFIKER RETAINING WALLS WITH FULL HEIGHT CAST-IN-PLACE PANELS

Construction Report

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206 South 17th Avenue
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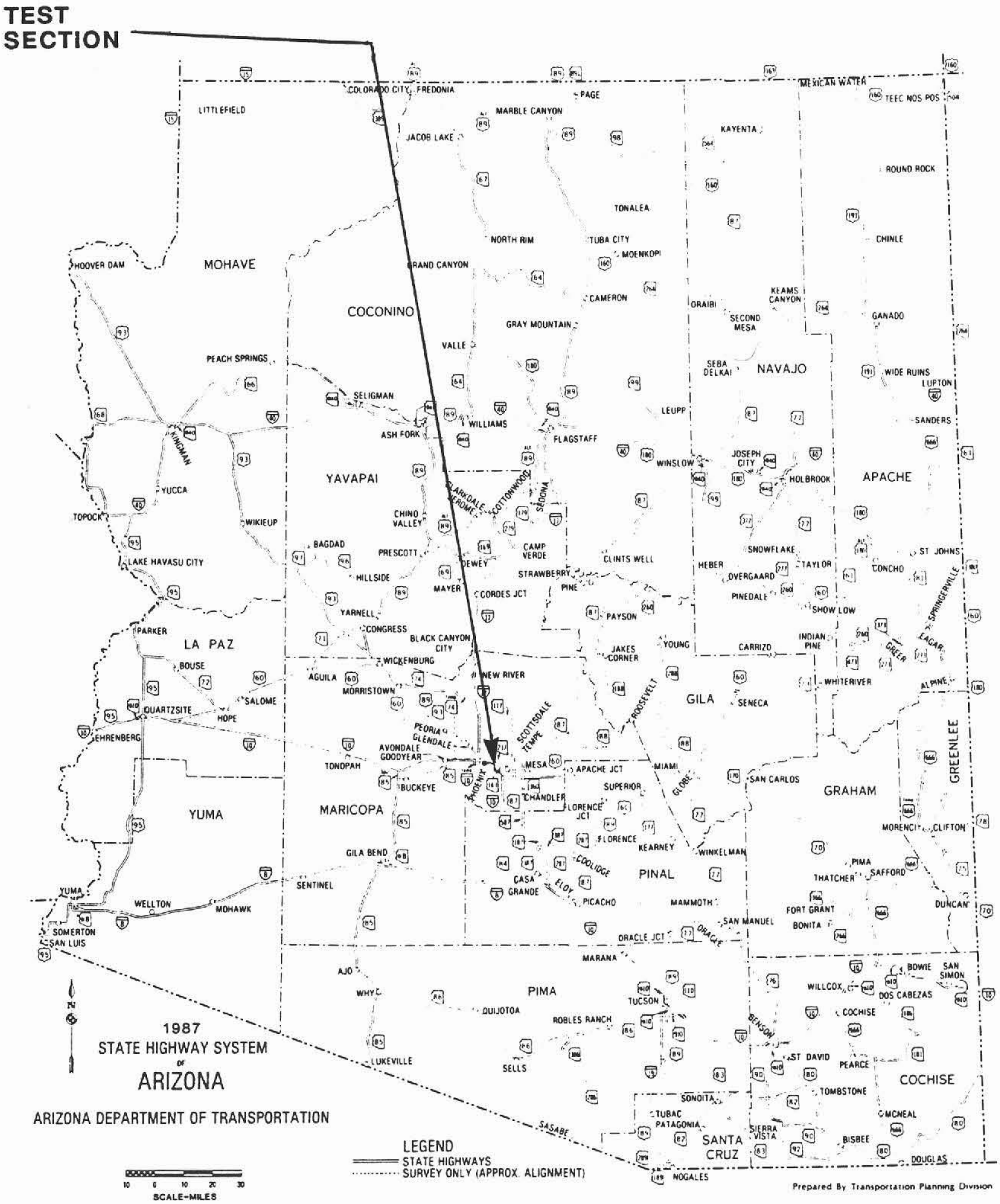

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16. ABSTRACT

Construction of retaining walls utilizing full height panels introduces a degree of indeterminacy to the structure. The ability to effectively analyze the internal behavior of such systems, and limited field performance, qualifies them as appropriate to the experimental category.

Arizona recently utilized this technique for building three permanent earth retaining structures at the interchange of I-10 and 24th Street in Phoenix, Arizona. The construction technique chosen was the Hilfiker Reinforced Soil Embankement with cast-in-place concrete facing.

The construction of the N.E. Wall started in September 1986 and finished in January 1987, while the construction of the S.W. Wall started in October 1986 and finished in November 1987. The construction of the S.E. Wall started in August, 1987, and finished in December 1987.

Settlement and movement of the walls are monitored by surveying bench marks embedded in the walls caps. Settlement up to 0.65" was measured in the N.E. Wall with a maximum lateral movement of 1.1" after three months of construction.

Hairline cracks and air pockets are visible in the finished sections of the concreted panels for these walls.

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I. Introduction

A. Background and Problem Statement

Mechanically Stabilized Embankment is a construction material composed of soil fill, which, for certain gradation and compaction conditions is strong in compression and shear, and of reinforcements such as rods, bars, fibers and geotextiles which are strong in tension. These inclusions (reinforcements) interact with soil by means of frictional resistance.

The basic idea of increasing the strength of soil by inclusions is not new. This has been done through centuries in several ways. Straw is usually added to adobe brick to increase its tensile strength. In the past, tree trunks and branches were used with soil for construction of dykes.

As mentioned above, the increase in strength of mechanically stabilized embankments is due to shear bond developed between the reinforcing strips and the soil grains. However, the shear bond is dependent on the effective vertical stress at any given depth. In cohesive soils e.g. clays and silts, due to lack of rapid drainage, the effective normal stress will vary depending on the degree of saturation and the pore water pressure. This makes the design somewhat unpredictable.

For that reason granular soils are usually used for construction of mechanically stabilized structures. Granular soils, e.g. sand and gravel, are free draining and hence, any increase of pore water pressure can be dissipated quickly.

The overall stability of a mechanically stabilized structure should include considerations for sliding, overturning, foundation bearing capacity and excessive settlement. Essential considerations for internal stability are that the reinforcements be safe against both pullout and rupture, and against reduction in effective area of reinforcement due to any means of deterioration such as corrosion.

In the last ten years the proliferation of proprietary mechanically stabilized retaining systems has left State and Federal Engineers with a complex problem of weighing the potential for significant initial cost savings against a frequently unknown product performance record.

Considerable experience has been gained in assessing the performance of these systems which typically utilize small modular precast facing panels approximately 25 sq.ft. in area. These panels, individually attached to the ground reinforcing elements, allow structures to incur considerable settlement without any significant distress.

In recent times, however, aesthetic considerations have compelled designers to require full height panels in lieu of the smaller modular panels.

The utilization of full height panels require the connection of multiple reinforcing elements to a single panel, introducing a degree of indeterminacy to the structure. The inability to effectively analyze the internal behavior of the systems and the paucity of satisfactory performance data, qualifies these systems as appropriate to the experimental category. Considerable concern exists as to the ability of these newer systems to sustain settlements.

B. Objective

The Federal Highway Administration had requested that three permanent earth retaining structures designed for project I-10-3(204) at the interchange of I-10 and 24th street in Phoenix, Arizona be classified as experimental projects.

This request was made because the full height face panels utilized in the retaining wall designs are considered experimental due to limited knowledge and experience with these systems. The three different wall systems listed below were approved by the Arizona Department of Transportation and it was up to the contractor (The Tanner Companies) to decide on which system to choose:

1. Reinforced Earth System with precast concrete face panels and cast-in-place coping

2. Retained Earth System with precast concrete face panels and cast-in-place coping

3. Hilfiker Reinforced Soil Embankment with cast-in-place concrete facing.

The contractor chose the Hilfiker system.

It is the objective of the Arizona Transportation Research Center to document the construction and evaluate the performance of the full height panel retaining systems for a period of at least three years (copy of the workplan is in Appendix A).

This construction report describes the method of construction, progress and difficulties encountered for the N.E. Wall (Ramp 24 A), the S.W.Wall, and the S.E. Wall (Ramp 24 B).

II. Project Location and Description

Project I-IR-10-3(204) begins at 16th street (I-17 milepost 195.09) and extends easterly to 28th street (milepost 150.39) for a distance of approximately 1.73 miles. Among the work included for this project is the construction of a new traffic interchange with Interstate Route 17 which includes grading, draining and placing portland cement concrete pavement.

The work consisted of constructing three permanent retaining walls at the intersection of I-10 and 24th street. These walls will be referred to in this report as: N.E.Wall which corresponds to "Hilfiker RSE Wall Ramp 24-A" as referred to in the plans; S.E.Wall which corresponds to "Hilfiker RSE Wall Ramp 24-B" as referred to in the plans; and the S.W. Wall which corresponds to "EB I-17 Hilfiker Reinforced Soil Wall" as referred to in the plans. See location map in Appendix B.

The three walls were constructed according to the Hilfiker Reinforced Soil Embankment System. Appendix C shows a copy of the cast-in-place (C.I.P.) construction guide provided by Hilfiker.

III. Plan and Profile

Copies of the plans and profiles for the three walls are provided in Appendix D.

IV. Soil Conditions

Four test pits and two drilled borings were performed along the alignment of the S.W.Wall (landfill area) as part of the geotechnical investigation during the design phase. Appendix E contains the boring data.

The excavation log of test pit number 1 showed gravelly sand and fill for the upper 2 ft, and sand, gravel, cobbles and some boulders between 2 ft and 11 ft. The excavation stopped at 11 ft. due to severe caving of material below 3 ft..

The excavation log for test pit number 2 showed gravelly sand and fill to 2 ft., followed by sand, gravel, cobbles, massive concrete blocks (5-6 ft. wide and 6-10 inches thick), asphalt, wood and metal scrap down to 18ft. The excavation was stopped at 18 feet due to severe caving. The concrete rubble may be indicative of a prior contractor's landfill.

Test pit number 3 showed gravelly sand and fill to 2 ft., concrete blocks with asphalt, wood, metal, sand, gravel, cobbles and boulders to 14 ft. and sand, gravel, cobbles, boulders to 19 ft..

Test pit number 4 showed gravelly sand and fill to 2 ft; concrete blocks with asphalt, wood, metal scrap, sand, gravel, cobbles and boulders to 8 ft. and sand, gravel, cobbles, boulders to 13 ft..

In boring number 84-135 silty sand with clay was found at the top 1 ft. while gravel and cobbles existed to 3 ft.. Refusal was at 3 ft. using the CME 55 auger.

In boring number 84-136 gravelly sand and fill was encountered to 2 ft.; landfill consisted of debris mixed with sand, gravel, cobbles and boulders existed to 8 ft., and sand, gravel, cobbles and some boulders existed to 13 ft..

In general the soil profile along the S.W.Wall alignment could be classified as poorly graded sandy material in the upper 2 to 3 feet and poorly graded gravelly material at lower depths with some organic material and concrete rubble present between the two layers. Boring data for the N.E.Wall and for the S.E.Wall was not available.

V. Embankment Construction

Preparation of the subgrade was performed by excavating to the design level. The soil was then tested for resistivity. If the material passed the resistivity test the construction continued; otherwise, the excavation continued for one more foot. In an isolated area additional excavation was performed in order to stabilize the excavation. An approved aggregate base course (ABC) was then spread over the over-excavated areas to the design level.

A reinforced concrete leveling course of fifteen inches in width and six inches in depth was constructed as a base for the cast-in-place concrete face of the Hilfiker wall (Figure No. 1). The final grade of the subgrade was sloping 1% to 2% away from the face of the Wall.

Figure No. 1 Preparation of the Subgrade and Levelling Course

The ABC material was placed and compacted using Caterpillar's 980C front loader and 140G blade. Water was sprayed from a 3.8 thousand gallon water truck after the ABC material was spread.

The first layer of reinforcing mats was placed on the subgrade and pinned in place with form pins (Figure No.2). Mats were made of galvanized welded steel wire mesh (W7xW7). The reinforcing (horizontal) mats were 7.5'x20'(variable), while the backing mats were 8'x2'. Figure Nos. 3 and 4 show typical dimensions of the reinforcing mats and the backing mats respectively.

The first lift of mat erection required a form "spacer cage" chair to hold the backing mat away from the vertical bars of the reinforcing mat (figures 8 and 9 of the C.I.P. Construction Guide -Pages C6 and C7).

Figure No. 2 Placement of the First Layer of Reinforcements

Behind the construction mats a woven geotextile fabric screen was placed in order to retain the compacted backfill behind the wire mesh (Figure No.5). The fabric was initially rolled under the mats but that left a 1/2" to 3/4" crevice which caused concern that the concrete might not fill the crevice. The fabric was then cut at 1/2 ft. intervals and placed in the vertical face of the mats.

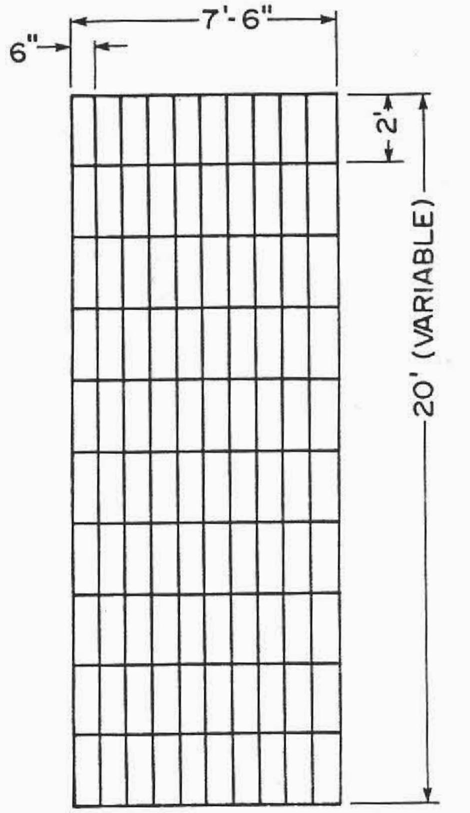
Every 2' lift a #4 non-galvanized rebar was placed, as per the plans, to control any expansion or contraction resulting from temperature changes (Page D3-Appendix D).

The backfill material was then end dumped on top of the reinforcements using a ten wheel dump truck. A grader (Caterpillar's 140G) leveled the backfill. Dumping and leveling continued until a 1' level of backfilling was reached, after which compaction was carried out.

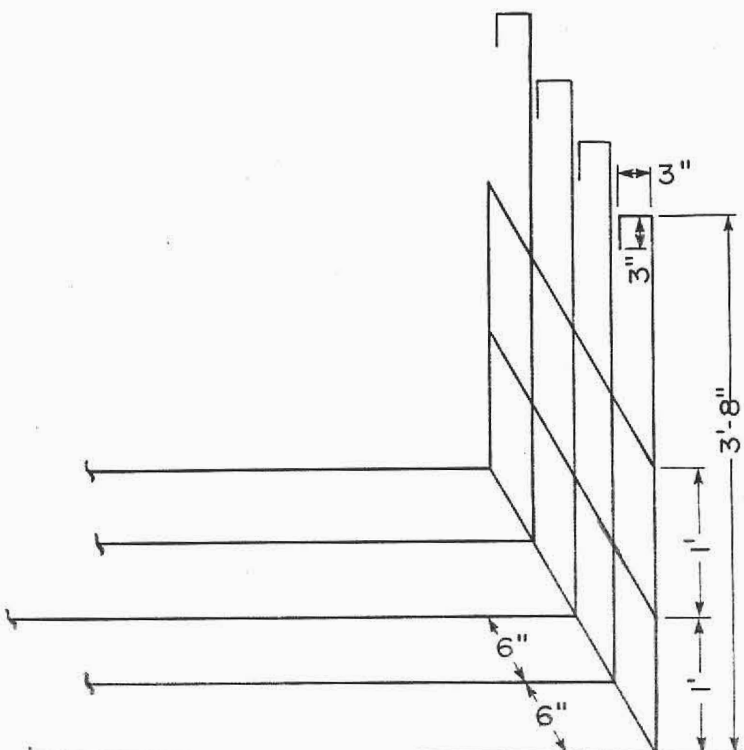
Compaction was performed by random passes of a rock-bucket loader, a grader, a small excavator-compactor backhoe loader, water trucks and the trucks which carried the backfill material to the site.

Compaction within 2 ft. nearest to the backing mats (2 foot zone) was performed by water jetting the 1 ft. closest to the backing mat, while the second foot was mechanically compacted by a hand vibrating tamper (jumping jack). Initially hand rammers were used but were found to be slow and inefficient. (Figure No. 6 shows the trucks used for dumping backfill material and the tampers used to compact the AB material in the 2 foot zone).

No special requirements were specified for the backfill material in the 2 ft. zone. However, attempts were made by the contractor to use backfill material that passed a 4" sieve size during the

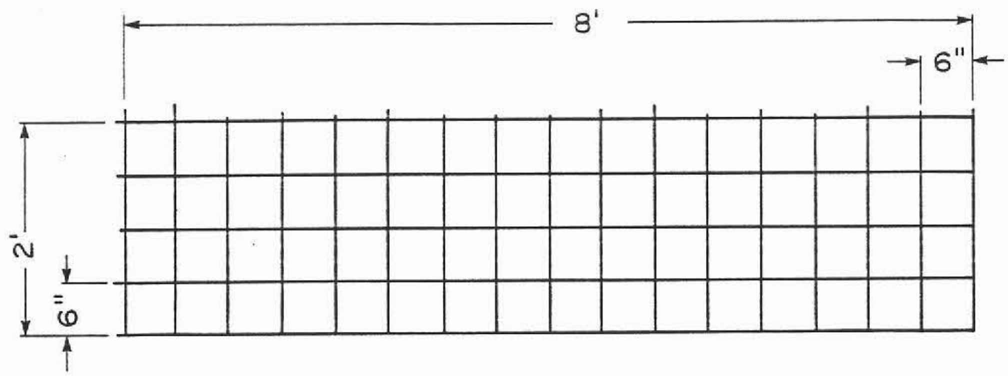


REINFORCEMENT MAT PLAN VIEW

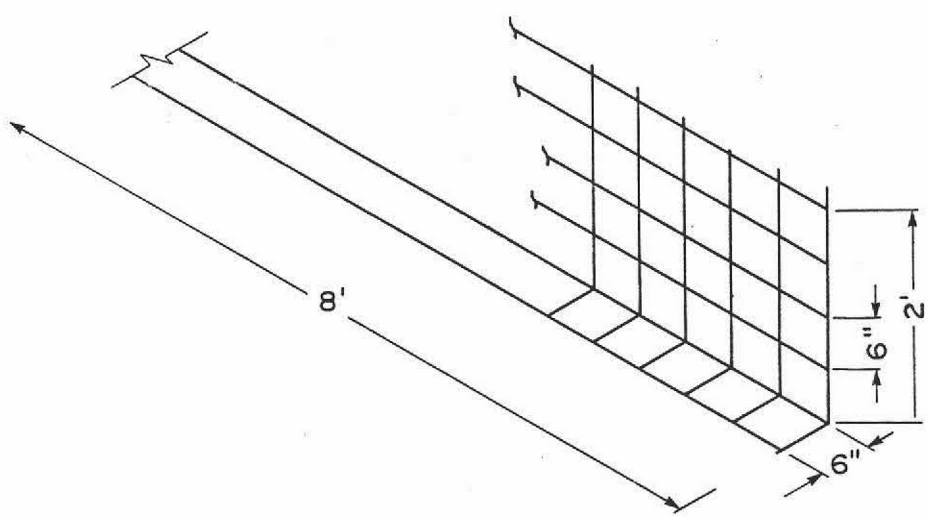


REINFOREEMENT MAT ELEVATION VIEW

Figure No. 3 Typical Dimensions of the Reinforcement Mats



BACKING MAT PLAN VIEW



BACKING MAT ELEVATION VIEW

Figure No. 4 Typical Dimensions of the Backing Mats construction of the N.E. and the S.W. walls. Such attempts, as witnessed by the author, did not always prove to be successful as can be seen in Figure No. 7.

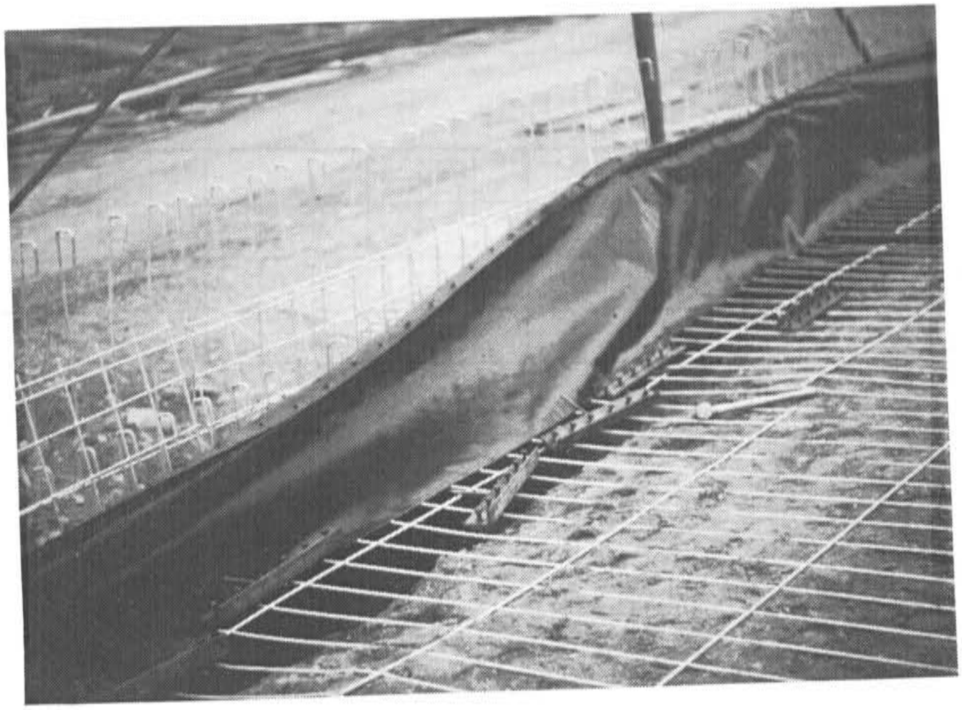


Figure No. 5 Tying of the Woven Geotextile Fabric Screen to the Reinforcements

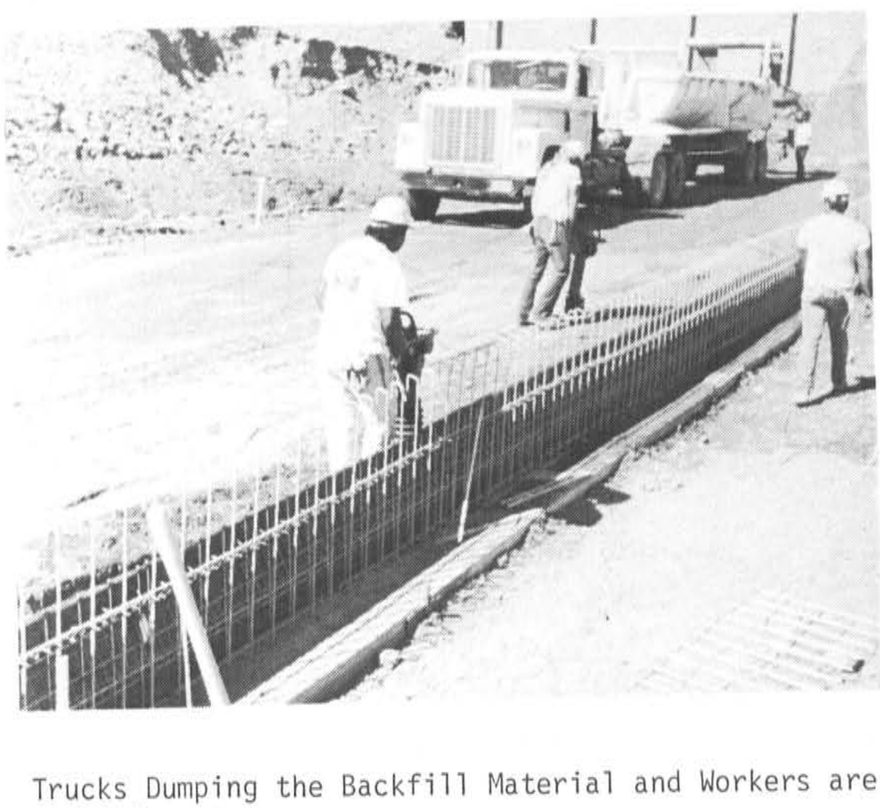


Figure No. 6 Trucks Dumping the Backfill Material and Workers are Compacting the 2 Foot Zone Area

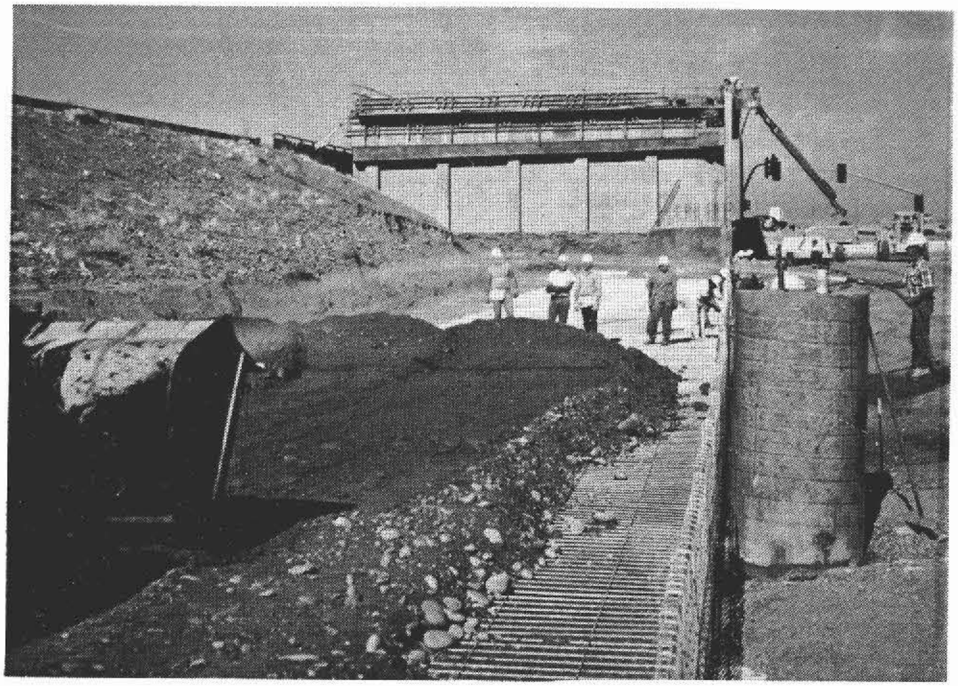


Figure No. 7 Backfill Material Used With Unsuccessful Attempts to Dispose of Rocks Larger Than 6" Sieve Size (Different Locations).

The backfill material for the N.E.Wall and most of the S.W.Wall consisted of a river run material obtained from the old embankments that existed on the site. This material had many rocks larger than 6". Attempts were made using a rock bucket and sometimes manual labor, to remove rocks that were larger than 6" sieve size. However, such attempts, as witnessed by the author, were not successful as 5% to 15% rocks larger than 6" sieve size were visible. Water was sometimes sprayed on the backfill material to enhance compaction (Figure No. 7).

Originally the backfill material obtained from the site was processed on a grizzly, and then mixed together in order to obtain the specified gradation. However, this material did not pass the minimum resistivity requirement of 3000 ohm-cm. Resistivity test results revealed failing resistivities. Results ranged from 536 ohm-cm to 2412 ohm-cm.

The backfill material for the S.E.Wall and a small section of the S.W.Wall consisted of a cohesionless material passing a 3" sieve size. This material was produced by Calmat of Arizona and had passed the specified resistivity test. Another backfill material produced by Union Rock company was also approved in case the backfill material produced by Calmat of Arizona was not sufficient.

Chloride, sulfate and pH tests were also performed on the backfill material to check its suitability against facilitating corrosion when in contact with the galvanized steel reinforcing mats.

Resistivity test results ranged from 3183 to 10300 ohms-cm, while the pH test results ranged from 7.9 to 9.3. Results from chloride and sulfate tests showed an absence of both. Appendix F contains a copy of ADOT's specifications for acceptance of backfill material, and Appendix G shows copies of typical results of resistivity, pH, chloride and sulfate tests for backfill material obtained from the river run on-site material and from Calmat of Arizona.

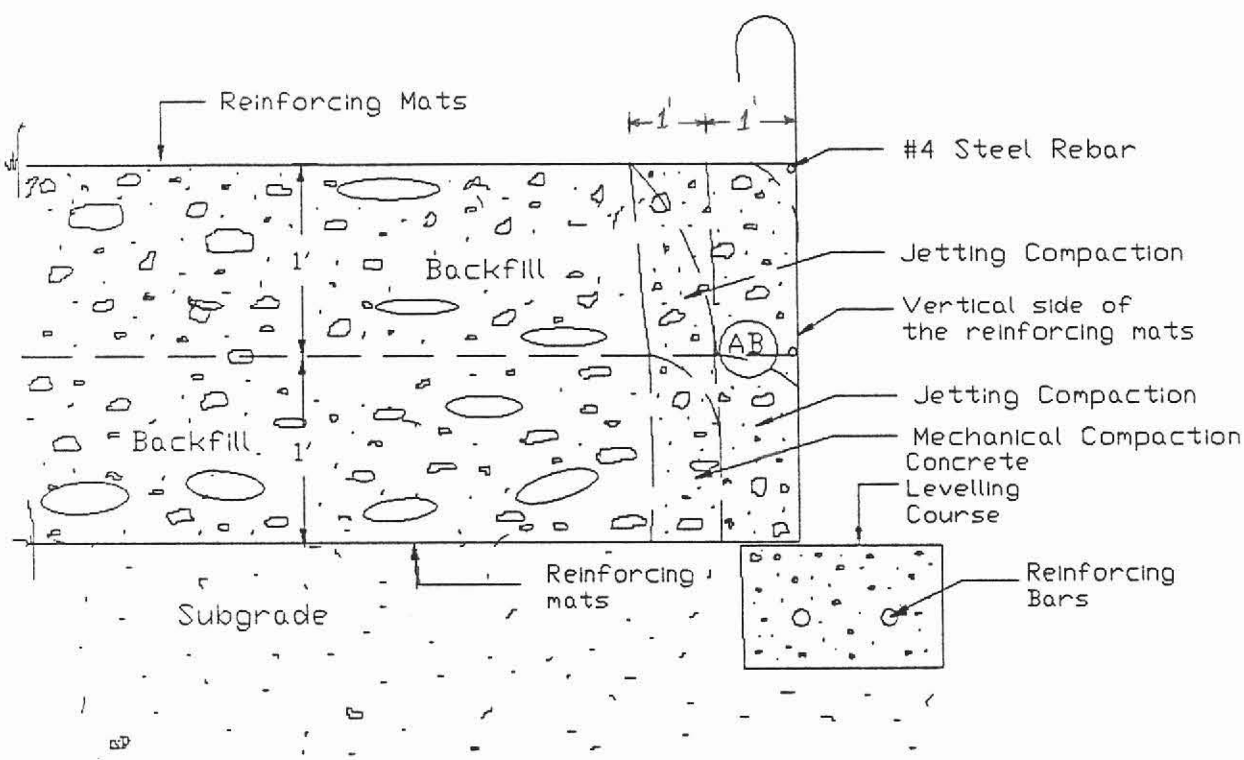


Figure No. 8 Schematic Diagram of a Typical Section of the Hilfiker Wall Constructed at the N.E. and the S.W. Walls

Random field density tests (AASHTO T-180), conducted by ADOT, showed that the specified 90% of the maximum dry density after compaction was achieved; field density test results ranged from 90% to 101%. Appendix H contains copies of typical results for field density tests.

The process of dumping, leveling and compacting the backfill material was repeated for the next 1' level after which a layer of reinforcement was placed. (Figure No.8 shows a schematic diagram of a typical section of the Hilfiker wall).

The contractor checked the outside vertical and horizontal alignments of the reinforcements by using a four foot carpenter's level and string line. With the addition of vertical layers of reinforcing mats the new mats lock into the proceeding ones. This locking effect ensured the verticality as the height of the wall increased (Figure No.9).

Figure No. 9 Workers Checking the Vertical and the Horizontal Alignment of the Hilfiker Wall

The area closest to the bridge abutment required that the structural backfill and the wall backfill should meet the required specifications of both backfills in the overlap areas. Page F4 shows a copy of the gradation for class 2 aggregate base that was used as a structural backfill, and pages F5 and F6 show a copy of the specifications for the structural backfill.